Chapter 2
Behavior of Embankments and Abutments

2-1. Introduction

A basic familiarity with geotechnical concepts is necessary to understand the application of instrumentation devices and analyses of instrumentation data. Key geotechnical aspects of embankment dam and levee behavior are described in this chapter. These geotechnical aspects are related primarily to pore water pressure and deformation. For a detailed treatment of these geotechnical topics, the reader is referred to other engineer manuals and geotechnical textbooks (e.g., Terzaghi and Peck 1967, and Holtz and Kovacs 1981). For detailed information on design and construction considerations refer to EM 1110-2-2300 and EM 1110-2-1913.

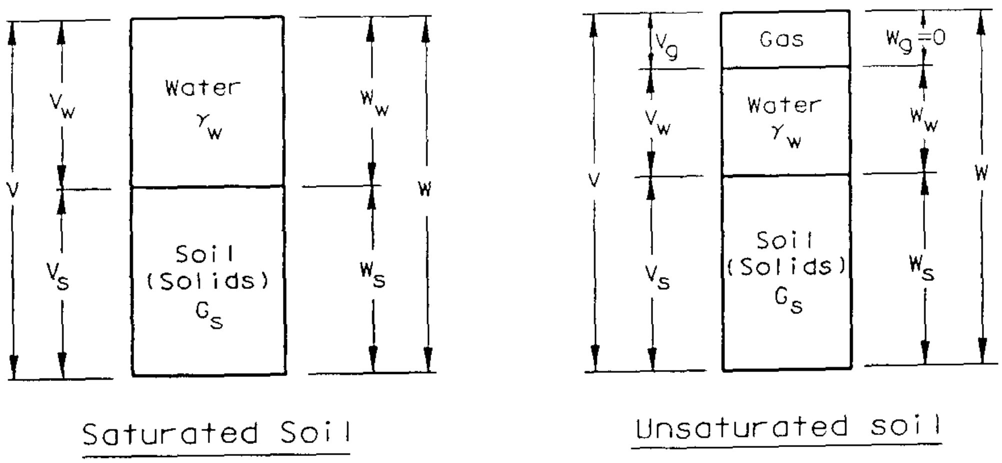
2-2. Basics of Soil Structure

a. Soil structure. Soil as it exists in the ground or as compacted in embankment fills consists of solid particles between which are spaces called ***voids, pores***, or ***pore spaces***. The pore spaces may be filled with either gas, usually air, or water. When all the voids are filled with water, the soil is ***saturated***. If any gas is present in the pore spaces, the soil is called ***unsaturated***. The term ***partially saturated*** or vadose zone is used by some when referring to unsaturated soils. Diagrams of both saturated and unsaturated soil structure are shown in Figure 2-1.

b. Types of soils. Soils are categorized into two broad groups: ***cohesionless soil*** and ***cohesive soil***. Cohesionless soils are those soils that consist of particles of rocks or minerals having individual grains usually visible to the eye. These materials, sands and gravels, are granular and nonplastic, meaning that they do not stick together when unconfined, and have little or no strength when air dried. Usually these soils have not been altered by chemical decomposition. Inorganic silt is fine grained with little or no plasticity and is considered a cohesionless material. Cohesive soils such as clays consist of fine-textured materials with microscopic and submicroscopic particles resulting from chemical decomposition of rocks. These materials have some strength when unconfined and air dried. Water affects the interaction between the mineral grains and gives plasticity or cohesion between the particles.

c. Stress and pressure. In geotechnical engineering, the terms ***stress*** and ***pressure*** are defined as force per unit area, with typical units of pounds per square inch ($lb/in.^2$) or pascals (Pa). Usually, pressure is a general term for force per unit area and stress is the force per unit area that exists *within* a mass. In the practice of geotechnical engineering, these terms are sometimes used interchangeably. The total force applied to or acting within a given area is known as the ***total stress***. That component of the total stress that is transmitted by grain-to-grain contact within the soil mass is called the ***effective stress***. The component of the total stress transmitted through the pore water is called the ***pore water pressure***, or the ***neutral stress***. The total stress consists of the sum of the effective stress and the pore water pressure. This relationship is known as Terzaghi's principle of effective stress. In unsaturated soils, a component of the total stress is transmitted to the pore gas portion of the voids and is called the ***pore gas pressure***. If pore gas pressure exists it will always be greater than the pore water pressure (Dunnicliff 1988). All the components of the total stress are applied over the same area as the total stress.

d. Consolidation. When a load is applied or increased on saturated soil layers, the increased force (total stress) is initially carried by the pore water pressure. The process of transferring the total stress to effective stress and decreasing the pore water pressure by squeezing out the water is known as ***consolidation***. Figure 2-2 illustrates what happens to the pore water pressure, effective stress, and volume of the soil as a total stress is applied. The change in volume shown in Figure 2-2b, that occurs as the total stress is transferred from the pore water pressure to the effective stress, generates vertical ***deformation*** that results in ***settlement***. The amount by which the pore water pressure exceeds the equilibrium pore pressure is known as the ***excess pore water pressure***. The decrease of the excess pore water pressure is called ***dissipation*** and is a function of the ***permeability*** of the material. Settlement results from the dissipation of excess pore water pressure and reorientation of the soil particles due to the additional load. Permeability is a measure of the rate at which water can move through the soil and determines how rapidly settlement occurs. When a soil has never been subjected to an effective stress greater than the existing overburden pressure, the soil is ***normally consolidated***. A soil that has been subjected to an effective stress greater than the existing overburden pressure is an ***overconsolidated soil***. Soils that have been subjected to glacier loadings are generally overconsolidated soils.



W = total weight of mass

W_s = weight of solids material

W_w = weight of liquid

W_g = weight of gas, usually assumed to negligible

V = total volume of mass

V_s = volume of solids material

V_w = volume of liquid

V_g = volume of gas

V_v = volume of voids = $V_w + V_g$ = volume not occupied by solids

G_s = specific gravity of solids

γ_w = unit weight of liquid

Figure 2-1. Saturated and unsaturated soil structure

Movement of the soil to resist sliding can also generate excess pore water pressure. The effective stress is related to the ability of the soil to resist sliding and its ***shear strength***. Thus, gains in shear strength during the consolidation process can be monitored by measuring pore water pressure. An example of this is the monitoring of pore water pressure as levees are constructed on soft foundations.

2-3. Groundwater Level and Pore Water Pressure

a. Hydrostatic piezometric conditions. The ***groundwater level*** or table is defined as the elevation that the free water surface assumes in permeable soils or rock because of equilibrium with atmospheric pressure in a hole extending a short distance below the capillary zone, as shown in Figure 2-3(a). The water-bearing stratum

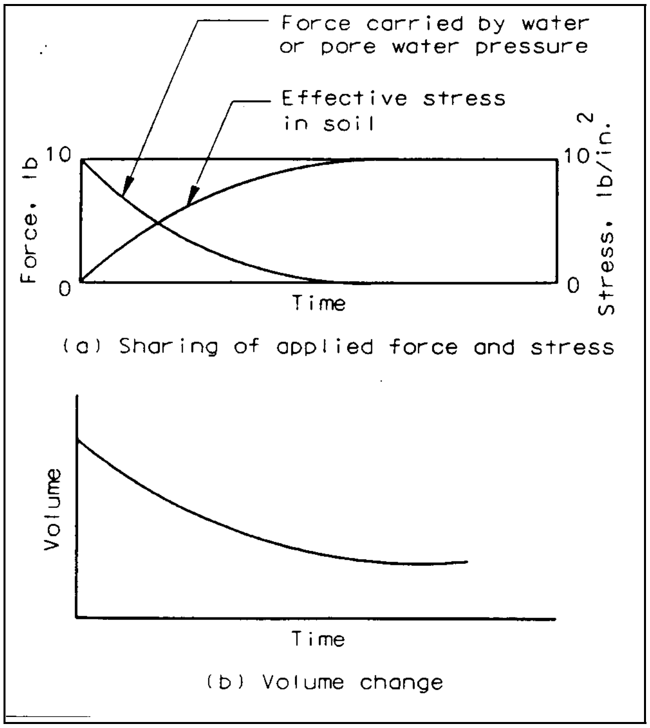


Figure 2-2. Response of effective stress, pore water, and soil volume to an applied total stress (Dunnicliff 1988)

containing the groundwater is called an ***aquifer***. The ***capillary zone*** is defined as the interval between the free water surface and the limiting height above which water cannot be drawn by capillarity. Normal behavior of groundwater conditions occurs when the pore water pressure increases hydrostatically with depth below the groundwater level. In this condition the pore water pressure, also called the ***hydrostatic pressure***, can be calculated by multiplying the unit weight of water by the vertical distance from the point of interest (screen location, sensor location, etc.) to the groundwater surface.

b. Pore water pressure. Figure 2-3 illustrates the groundwater condition soon after a layer of material is placed over existing soil layers and before consolidation is complete. Thus, excess pore water pressure exists in the clay and the groundwater is no longer in equilibrium. The five perforated pipes in Figure 2-3 are installed such that the soil is in intimate contact with the outsides of the pipes. The second pipe (pipe b) is perforated throughout its length while the other pipes are perforated only near the bottom. Due to the high permeability of the sand, the excess pore water pressure in the sand is dissipated immediately. Thus, pipe (a) indicates the groundwater level. Pipe (b) indicates the groundwater level because the permeability of the sand is such that the excess pore water

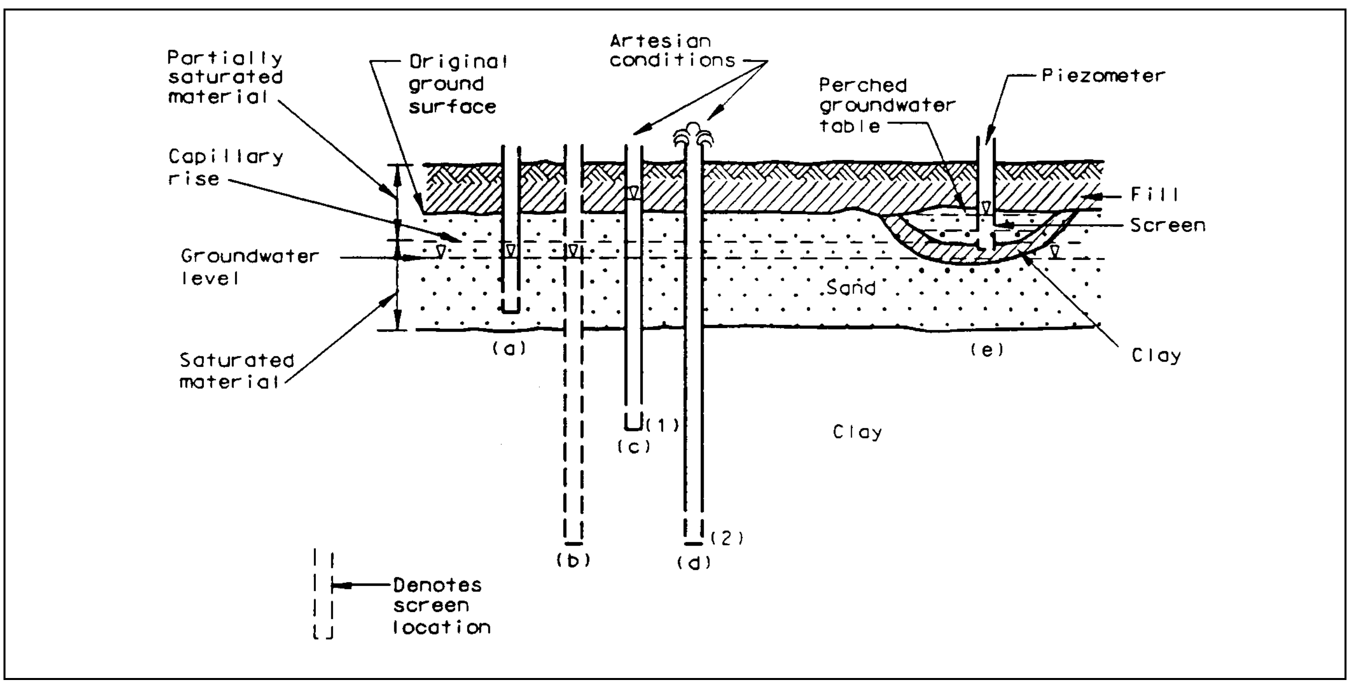


Figure 2-3. Groundwater level and pore water pressure when there is flow of groundwater (Dunnicliff 1988)

pressure from the clay will be dissipated into the sand. Pipes (c) and (d) indicate the pore water pressures in the clay at two locations. More dissipation of the excess pore water pressure has occurred in pipe (c) than in pipe (d) because the drainage path for excess pore water pressure is shorter and the rate of dissipation is greater in pipe (c) than in pipe (d). Pipe (b) in Figure 2-3 is an ***observation well***, because there are no subsurface seals that prevent a vertical connection between multiple strata. A ***piezometer*** is a measuring device that is sealed within the ground so that it responds only to groundwater pressure around itself and not to groundwater pressure at other elevations. Pipes (a), (c), and (d) are called piezometers because they indicate pore water pressure at one location (they are sealed above and below the perforated locations) and not to the groundwater pressure at other elevations. The ***piezometric elevation*** or ***piezometric level*** is the elevation to which water will rise in a piezometer.

c. Positive pore water pressures. Pore water pressure that is above atmospheric pressure is called ***positive pore water pressure***. Pore water pressure can be increased by applying a compressive force to the soil or when a shearing force is applied to a soil that decreases its volume while preventing dissipation of pore water pressure. Excess pore water pressure resulting from any type of stress change can also be called ***induced pore water pressure***.

d. Negative pore water pressures. ***Negative pore water pressure*** occurs when the pore water pressure is less than atmospheric pressure. This condition can occur when a compressive load is removed or when a densely packed soil is sheared and increases in volume.

e. Non-hydrostatic groundwater conditions. Pore water pressure does not always increase hydrostatically with depth below the groundwater level. These groundwater conditions include:

(1) Perched water table. ***Perched water tables*** are caused when a permeable material overlies a relatively impermeable strata above the main groundwater level and retains some groundwater. A piezometer placed in a perched water table will indicate an elevated surface as illustrated in Figure 2-3, pipe (e).

(2) Artesian pressure. ***Artesian pressures*** are found in strata that are confined between impervious strata and are connected to a water source at a higher elevation. A well drilled to an artesian aquifer having a pore water pressure above the ground surface will flow without pumping and is called a free-flowing artesian well. Artesian conditions are shown in Figure 2-3, pipes (c) and (d).

f. Variation in piezometric levels and pressures. Piezometric levels and pressures are rarely constant over an extended period of time. Natural forces such as precipitation, evaporation, atmospheric pressure, and seepage may cause wide variations in the groundwater level.

2-4. Time Lag in Groundwater Observations

a. Hydrostatic time lag. Most piezometers require some movement of pore water to or from the device to activate the measuring mechanism. When pore water pressures change, the time required for water to flow to or from the piezometer to create equalization is called the ***hydrostatic time lag***. The hydrostatic time lag is dependent primarily on the permeability of the soil, type and dimensions of the piezometer, and the change in pore water pressures. The volume of flow required for pressure equalization at a diaphragm piezometer is extremely small, and the hydrostatic time lag is very short. For an open standpipe piezometer, the time lag may be reduced by providing a large intake area and reducing the diameter of the riser pipe, thereby reducing flow required for pressure equalization. The pressure of gas bubbles (natural or result of corrosion) can also cause a time lag. Hydrostatic time lag is not significant when piezometers are installed in highly pervious soils such as coarse sands. Those piezometers that have a long time lag are described as having a ***slow response time***. The time required to establish equalization of pore water pressures after installing or flushing a piezometer is called the ***stress adjustment time lag***, sometimes referred to as the ***installation time lag***.

b. Determination of time lag. An estimate of the hydrostatic time lag aids in selecting the proper type of piezometer for given subsurface conditions at a given site. The order of magnitude of the time required for 90% response of several types of piezometers installed in homogeneous soils can be determined from Figure 2-4. As stated in Dunnicliff (1988), the 90% response is considered reasonable for most practical purposes since the 100% response time is infinite. The response time of open standpipe piezometers can be estimated from equations derived by Penman (1960). As an example,

$$t = 3.3 \times 10^{-6} \frac{d^2 \ln \left[L/D + \sqrt{1 + (L/D)^2} \right]}{kL} \qquad (2\text{-}1)$$

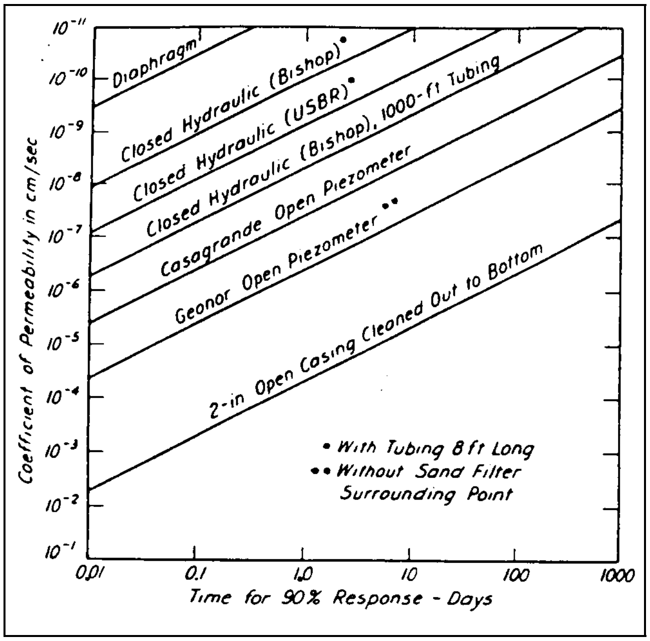


Figure 2-4. Approximate response time for various types of piezometers (after Terzaghi and Peck 1967)

where

t = time required for 90% response, days

d = inside diameter of standpipe, centimeters (cm)

L = length of intake filter (or sand zone around the filter), centimeters (cm)

D = diameter of intake filter (or sand zone), centimeters (cm)

k = permeability of soil, centimeters per second (cm/sec)

Similar equations and other procedures have been developed by Hvorslev (1951) and others. It is often more desirable to measure time lag in the field by comparing pool fluctuation with desired piezometer readings as illustrated in Figure 2-5.

2-5. Mechanisms That Control Behavior

The engineering behavior of soils is controlled by either hydraulic, stress-deformation, or strength mechanisms. When a soil is subjected to excess pore water pressures, the water flows through the pore spaces creating friction and a resistance to flow. The permeability of the soil is a

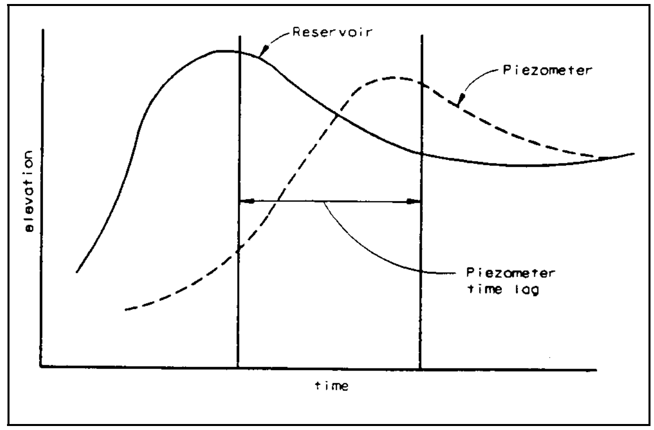


Figure 2-5. Time lag measurement from time-history plot of reservoir level and piezometer

function of this friction and resistance to flow. When water is flowing through soil, the water exerts a force on the soil particles in the direction of flow. Also if the soil is saturated, water acts on the soil with a buoyant force whether or not the water is moving. Stress-deformation characteristics of cohesionless soils are caused by rearrangement of the relative positions of grains as shear deformation occurs. For cohesive soils, the stress-deformation characteristics are governed by the time required for water to flow through the pore spaces in the soil and for subsequent volume changes to occur. Shear strength is governed by the nature, size, and shape of the soil grains, packing density, and effective stresses within the soil. Shear failure occurs when the applied stresses increase beyond those that can be sustained by the soil.

a. Embankments and foundations. Earth and rockfill embankments may experience failure by the following mechanisms: overtopping, instability, and internal seepage. Overtopping results when the reservoir or river level exceeds the embankment height. This condition can occur due to incorrect estimations of water volume or as the result of some other mechanism (i.e., slope instability) causing a displacement of a large mass of water.

(1) Embankment and foundation instability. When the available shearing resistance along a potential surface of sliding of an embankment slope is greater than the shear stress or driving force on that surface a stable slope results. Any increase in pore water pressure along the potential surface of sliding decreases the shearing resistance and the factor of safety against sliding. If the foundation is stronger than the embankment soil, the slope will generally move within the embankment. If the

embankment is overlying a soft foundation, the properties of the foundation material will determine if stability is a problem. The embankment loading could cause a movement through the foundation or a movement along a weak foundation layer. In addition, the embankment could cause settlement and lateral bulging of the foundation. If the foundation consists of loose uniform cohesionless material, an earthquake can cause liquefaction and sliding within the foundation.

(2) Piping. Seepage can cause piping (internal erosion) when the velocity of the flowing water is sufficient to move or carry soil particles. If the soil has some cohesion, a small tunnel or pipe could form at the downstream exit of the seepage path. As the soil is eroded, the pipe lengthens inside the embankment. Once piping begins, there is less resistance to flow, the flow in the pipe increases, and piping accelerates. Silt and fine sands are most prone to piping. In a well-designed embankment, piping is prevented by using filters or drains to ensure that material from upstream cannot migrate downstream.

(3) Transverse cracking. The load of the embankment can cause foundation settlement and some settlement and lateral movement in the compacted fill. Uniform settlement of the embankment or foundation is usually not a problem. However, when the abutments are steep, settlement may cause cracking transverse to the axis of the dam.

b. Soil around structures and abutments. When soil is placed around structural elements or against abutments, behavior mechanisms of concern include seepage, stability, and sliding of the structural element. The interface between the soil and either the abutments or a structural element is a potential location for abnormal seepage. Special care and compaction effort are needed in this area to ensure that the potential for piping is minimized. In addition, whenever material is placed against abutments or other existing surface, the potential exists for weak shear surface planes to develop, which could become a shear surface. Good compaction along the interface should prevent this type of mechanism. Sliding of a structural element is a function of the force caused by the ***active earth pressure*** pushing on the element minus the shearing resistance along the base of the element and the force caused by the ***passive earth pressure***.